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HAROLD E. GRAY[3].—A problem which has confronted many farmers and other workers in the field of supplemental irrigation in the northeast, and which will continue to increase as the practice of irrigation spreads, has been presented by the author.

An indication of the magnitude of the problem can be obtained from information on the amount of irrigation being practiced in New York State. This information is based on the results of a survey conducted in 1950 by Harry A. Kerr. Reports from fifty-two counties in the state showed nearly two thousand farmers using supplemental irrigation in 1950. Although over one half of these were concentrated in the two counties on Long Island (New York), forty-nine of the fifty-two counties reported the use of irrigation. The rapid growth is indicated by the fact that this same survey showed that slightly more than one thousand farmers were using irrigation 5 yr prior to the time of survey and that only six hundred and fifty were using it 10 yr before. Furthermore, there were six hundred and fifty farmers who indicated that they were planning to install irrigation systems in the immediate future, probably by 1952. Unfortunately, no figures are available to confirm this last statement.

At present, irrigation is practiced on nearly all vegetables grown commercially and for home consumption. Small berries, strawberries in particular, have responded to the addition of supplemental water. There is an increasing interest in the use of irrigation on pasture by dairy farmers.

The general feeling among farmers who have used irrigation is that it will save a crop 1 yr out of 5, give increased yields 2 or 3 yr out of 5, and that it is useless the other 1 or 2 yr. However, much more information is necessary concerning the consumptive use of water for different crops in humid climates, and concerning drought frequency expectancy, before this condition can be stated as a fact.

On Long Island, where there are heavy concentrations of irrigated areas, the need for the control of water has already become apparent and steps have been taken to insure this control. A similar need has not been recognized in other parts of the state. It is conceivable that, with the increased use of water for irrigation, the situation will become critical. This problem needs considerable study so that a workable system of legally-controlled water rights can be established. No one can argue against the advantage of establishing such a control to forestall a critical situation, as opposed to the establishment of the control to correct a critical situation after it has occurred.

There has been considerable interest in new sources of water supply for irrigation. The author indicated the increase in the building of farm ponds in New York State as a result of drought. However, caution must be observed in recommending farm ponds as ready and economical sources of water for

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irrigation. A few figures will serve to emphasize this face. The farm pond of average size probably will hold 500,000 gal. It takes more than 27,000 gal to apply 1 in. of water on 1 acre of ground. A 1-in. application on 10 acres of ground would take more than one half the capacity of this average-sized pond. When the pond depends upon runoff for replenishment, especially during dry periods when the need for irrigation water is greatest, the supply of water may not meet the demand. Furthermore, this example makes no provision for pond losses, such as evaporation, seepage, and other uses of water.

The writer does not intend to assert that farm ponds cannot be used for irrigation. His intent was to indicate that, if ponds are to be used for irrigation, careful planning and design are essential to their success, so that an adequate supply of water will be available at the time of greatest demand.

In conclusion, it is repeated that, with increased demand for water for irrigation in the humid regions, a thorough study of the problem is essential in order to derive a workable system of legal control of water rights.

HOWARD T. CRITCHLOW,[4] M. ASCE.—Since preparing the original paper, the writer has participated in a survey of ground-water supplies for the United States. This survey was made in connection with a conference on water resources in Urbana-Champaign, Ill., in October, 1951. The significant fact disclosed by the survey, with relation to irrigation in humid areas, is that, of the twelve states having legal control over the allocation and use of ground water, New Jersey is the only state in which such control applies to irrigation and agricultural use. This law requires that diverters obtain permits from the state water control agency to use ground water in excess of 100,000 gal daily in certain areas of the state. To date (1952) this control has not restricted the development of supplemental irrigation, but there are indications that the extent of such irrigation may be limited by the availability of ground water.

The discussion by Mr. Gray indicates a rapid growth of supplemental irrigation in New York, largely from wells on Long Island and from surface supplies in up-state regions. One important fact emphasized by Mr. Gray is the danger of failure of the source of water supply from small ponds in drought, unless ample provision is made for evaporation, seepage, and other losses of water from such sources. Several cases have occurred in New Jersey, where irrigators have depended upon surface streams and ponds for irrigation use, only to have these sources fail in drought periods, with a resultant loss of valuable crops. The development of well supplies has resulted from such experiences. Good agricultural engineering practice is the only cure and guarantee that will assure successful irrigation, even in normally humid areas.

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AHMED SHUKRY,[7] A. M. ASCE.—The deviation of the free surface, in the vicinity of an unconfined well, from that computed by the Dupuit equation has long been recognized, although it is still ignored by some engineers. This deviation is generally attributed to the following two causes:

(1) The flow of water through porous soils obeys the Darcy law if the slope is less than $\frac{dh}{ds}$. For greater slopes, actual velocities will be smaller than those computed by this law. The limiting slope for the application of the Darcy law was found to depend on the permeability coefficient: According to E. Prinz,[8] this slope does not exceed 1 on 1 in ordinary sand.

(2) Two approximate assumptions that are used in the Dupuit equation produce deviation. They are the assumption that the slope of the free surface is $\frac{dh}{dr}$ instead of $\frac{dh}{ds}$, and the assumption that the potential surfaces are vertical cylindrical surfaces. Both assumptions nearly represent the actual conditions at some distance from the well and, for this reason, the measured seepage discharges agree closely with those computed according to the Dupuit equation. This agreement has been confirmed by different authors adopting diverse methods of investigation such as mathematical analysis,[3] electric analogy,[5] sand model tests,[5,9] and the relaxation method.[9]

Leo Casagrande,[10] when applying the Dupuit equation to seepage through earth dams, assumed the slope of the water surface to be $\frac{dh}{ds}$ instead of $\frac{dh}{dx}$. Applying the same modification for the case of the flow of unconfined ground water toward a single well, the basic differential equation becomes

$$Q = K\,2\,\pi\,r\,h\,\frac{dh}{ds} \qquad (21)$$

The slope of the water surface in Eq. 6 may be modified accordingly, so that the equation may represent more closely the actual conditions near the well.

Relative to the distribution of pressures at the impermeable base, the author's results confirm those obtained by Messrs. Wyckoff, Botset, and Muskat[4] who referred to the Dupuit equation as the "base-pressure equation." This conclusion was also checked by Messrs. Babbitt and Caldwell, who performed tests on electric and sand models.[5] On the other hand, using the relaxation method, S. T. Yang[11] found that the Dupuit equation does not

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[8] "Hydrologie," by E. Prinz, J. Springer, Berlin, Germany, 2d Ed., 1920.

[9] "The Flow Pattern Near a Gravity Well in a Uniform Water-Bearing Medium," by Norman Savage Boulton, *Journal*, Inst. of C.E., London, England, December, 1951, pp. 534–550.

[10] "Naeherungsmethoden zur Bestimmung von Art und Menge der Sickerung durch geschuettete Daemme," by Leo Casagrande, Technische Hochschule, Vienna, Austria, July, 1932. (Translated into English by staff, U. S. Waterways Experiment Station, Vicksburg, Miss.)

[11] "Seepage Toward a Well Analyzed by the Relaxation Method," by S. T. Yang, thesis presented to Harvard Univ., at Cambridge, Mass., in 1949, in partial fulfilment of the requirement for the degree of Doctor of Science.

yield the correct distribution of pressures along the impermeable base. In the writer's opinion, the agreement between the Dupuit equation and the piezometric base pressures is valid only at some distance from the well; but in the region where the free surface is sharply curved, a marked discrepancy should be expected between the two values. The writer checked this point from a pattern of flow toward an unconfined well, solved by Mr. Boulton[9] using the relaxation method (Fig. 11).

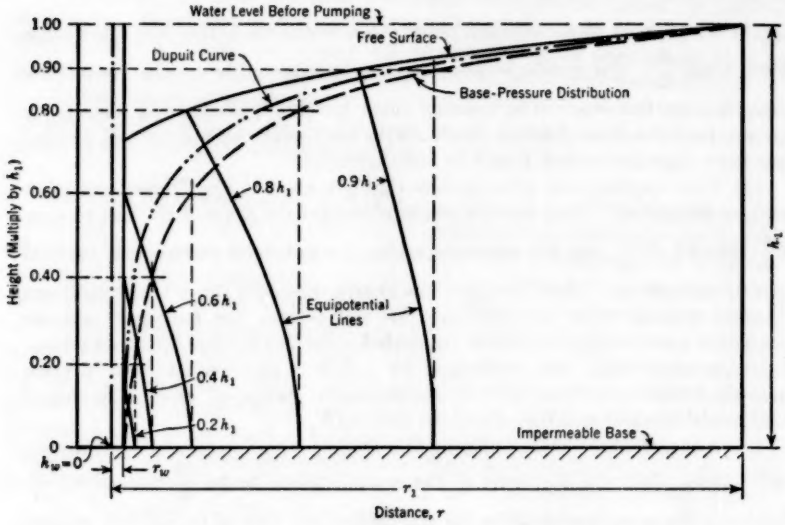

FIG. 11.—THE PRESSURE DISTRIBUTION AT THE IMPERMEABLE BASE AS OBTAINED FROM A FLOW PATTERN SOLVED BY THE RELAXATION METHOD

Eq. 8 is in an empirical form. To enable a clear study of the slopes of the experimental functions shown in Fig. 7, it is suggested to derive this equation from the Dupuit equation as follows:

$$h^2 - h_1^2 = \frac{Q}{\pi K} \log_e \frac{r}{r_1} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Therefore,

$$h - h_w = \frac{Q}{\pi K (h + h_w)} \log_e \frac{r}{r_w} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (22)$$

Therefore, the theoretical values of the slopes in Eq. 8, according to the Dupuit assumptions, would be given by

$$m = \frac{Q}{\pi K (h + h_w)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (23)$$

In this form, the experimental values of m can be compared with the Dupuit values. It has been stated that the free surface profile of the flow

agrees favorably with the Dupuit equation, at long distances from the well. According to Messrs. Babbitt and Caldwell[5] and Mr. Boulton,[9] this distance is approximately 1.5 h. Because the Dupuit value of the slope m as given by Eq. 23 is not constant, it follows that the logarithmic functions shown in Fig. 7 cannot be straight lines. However, they may be considered, approximately, as such for large values of r because of the flatness of the free surface. In the vicinity of the well, where the free surface curves down sharply, the values $(h + h_w)$ in Eq. 23 decrease considerably. For $h_w = 0$, the Dupuit value of m should be infinite at $\frac{r}{r_w} = 1.0$, and the theoretical functions as given by Eq. 22 should be vertical. With the departure of the free surface from the Dupuit curve occurring near the well, the corresponding curvature of the experimental function would be less. An interesting addition to Fig. 7 would have resulted if the free surfaces had been determined, for the four tests, by ordinates closer to the wells.

The constants C_1 and C_2 in Eq. 9 are not dimensionless (or at least one of them is not). To permit a study of the behavior of these constants with respect to the controlling parameters, the writer suggests the use of one dimensionless constant C whose theoretical value can be obtained as follows:

From Eq. 22,

$$h - h_w = \frac{Q}{2\pi K h_w} \frac{2 h_w}{h + h_w} \log_e \frac{r}{r_w}$$

or

$$h - h_w = \frac{Q}{2\pi K h_w} C \log_e \frac{r}{r_w} \quad \ldots\ldots\ldots\ldots\ldots\ldots (24)$$

Mr. Boulton[9] used an equation similar to Eq. 24, but fitting the Dupuit equation to the outer boundaries, as follows:

$$h_e - h = \frac{Q}{2\pi K h_e} \frac{2 h_e}{h_e + h} \log_e \frac{r_e}{r} \quad \ldots\ldots\ldots\ldots\ldots\ldots (25)$$

and writing—

$$\frac{2\pi K h_e (h_e - h)}{Q} = \xi$$

Eq. 25 becomes

$$\xi = \frac{2 h_e}{h_e + h} \log_e \frac{r_e}{r} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (26)$$

If r equals r_w, it follows that $h = h_s$ and $\xi = \xi_s$. Mr. Boulton investigated the experimental and the relaxation values of the dimensionless number ξ_s with respect to both the dimensionless parameters $\frac{r_w}{h_e}$ and $\frac{h_w}{h_e}$, which define the conditions at the well. An important finding of Mr. Boulton's work is that ξ_s, or rather the drawdown $(h_e - h_s)$ at the well, is practically independent of the water level in the well for ranges of $\frac{h_w}{h_e}$ between zero and 0.40. It would be interesting if the author would check this important conclusion, which can be proved also as follows:

From Eq. 21, $\frac{ds}{dh} = \frac{2\pi K}{Q} r h$. Assuming that $(ds)^2 = (dh)^2 + (dr)^2$ and referring to Fig. 12, $1 + \left(\frac{dr}{dh}\right)^2 = \frac{4\pi^2 K^2}{Q^2} r^2 h^2$. Letting $\frac{2\pi K}{Q} = b$, the equation can be written as follows:

$$\frac{dh}{dr} = \frac{1}{\sqrt{b^2 r^2 h^2 - 1}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (27)$$

Using the theoretical value of the slope of the free surface where it joins the well,[12] namely, 90°, and letting h equal h_s at $r = r_w$ and because $\frac{dh}{dr} = \infty$ at the well, $\sqrt{b^2 r_w^2 h_s^2 - 1} = 0$, from which

$$h_s = \frac{1}{b r_w} \quad \ldots\ldots\ldots (28)$$

Replacing b by its value, the following relationship is obtained:

$$h_s = \frac{Q}{2\pi K r_w} \quad \ldots\ldots (29)$$

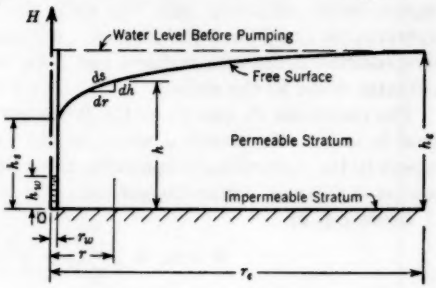

FIG 12.—FLOW TO AN UNCONFINED WELL

From this equation it can be seen that h_s is independent of h_w. This derivation, which is based on the modification of the water slope in the Dupuit equation, confirms both Mr. Boulton's results and the author's conclusion that the zone of seepage increases as the drawdown increases.

The author's discussion of the potential distribution through the capillary zone of a sand model is illuminating (see under the heading, "Model Studies of Unconfined Flow"). The main disadvantage of sand model tests which caused some investigators to recommend relaxation[9,11,12] methods or iteration[13] methods was the difficulty in locating the exact position of the free water surface.

The author should be complimented for developing functional relationships between the variables at the well. His statement (see under the heading, "Model Studies of Unconfined Flow," the paragraph preceding Eq. 10) that "Under ordinary field conditions there is no definite radius of influence because the conditions far from the well are dependent on the natural recharge ***" should be considered carefully by future investigators in their attempts to provide fruitful results for use by the practical engineering profession.

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[12] "Relaxation Methods Applied to Engineering Problems," by F. S. Shaw and R. V. Southwell, Chapter VII of "Problems Relating to Percolation of Fluids Through Porous Materials," *Proceedings*, Royal Soc. of London, Series A, No. 972, Vol. 178, May, 1941.

[13] "Numerical Solutions of Steady-State and Transient Flow Problems; Artesian and Water-Table Wells," by A. A. I. Kashef, Y. S. Touloukian, and R. E. Fadum, *Research Series No. 117*, Purdue Eng. Experiment Station, Lafayette, Ind., 1952.

CARL ROHWER,[14] M. ASCE.—Problems involving confined or unconfined flow of water through porous media are complicated in that, even in the laboratory, it is almost impossible to obtain a truly homogeneous material. This is because of the effect of entrapped air and the difficulty in getting uniformly compacted material. Another difficulty is encountered if the water passing through the material being tested is increasing in temperature. This results in the air in solution being liberated, causing a reduction in the effective pore space. Likewise, if the water temperature is reduced as it passes through the material, air will be absorbed and the permeability of the media increased.

Furthermore, the application of the hydraulic principles governing the flow of water into wells involves mathematical problems, the solution of which cannot even be approximated without making assumptions which affect the validity of the analysis. In spite of these difficulties, the author has obtained remarkably consistent results from his experiments and has obtained new relationships which should be of help in interpreting well-flow phenomena.

Of particular interest is Fig. 6. This illustrates that the Dupuit equation is reasonably accurate in defining the free water surface beyond the immediate vicinity of the well. Since the velocity of the water through the sand near the well is much greater than the velocity at the periphery of the model, it is possible that the flow through some of the larger interstices of the sand near the well may be turbulent. If this is true, this portion of the flow will follow Torricelli's law and, therefore, will affect the position of the free water surface and the shape of the potential lines. The curvature of the potential lines in the area near the well, as shown in Fig. 5, results in the development of the seepage face; and, since the seepage face is affected by the nature of the flow in this region, the change from laminar flow to turbulent flow will affect its height. However, this effect may be so small that it may not be possible to detect it in a model study.

According to Fig. 7, the plot of the adjusted free water surface, with h as the ordinate and $\log_e\left(\frac{r}{r_w}\right)$ as the abscissa, is a straight line for values of $\frac{r}{r_w}$ greater than 2. The author concludes from this fact that this line may be extended to the well to determine the elevation of the free water surface at the well. This procedure may give the correct result, but extrapolation is usually an unsound expedient. This is particularly true in the case under discussion because of the many uncertainties as to the nature of the flow in the zone adjacent to the well. It would have been helpful if the author had shown the actual elevation of the free surface at the well, as determined by the model study, even though precise readings at this point were difficult to obtain.

This comment is not intended to minimize the importance of the author's results, since Eq. 8 should be useful in checking the accuracy of piezometer readings and in locating the free water surface at the well. In field studies where the free water surface is at a considerable depth below the ground surface, the free surface near the well is difficult to determine because the location of the piezometer with reference to the well is uncertain. Eq. 8 provides a

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means of obtaining the elevation of the free surface in this zone if the elevations of points removed from this zone are known.

The author states that the dimensionless parameter, given in Eq. 11, indicates whether the well has a deep or shallow cone of depression. This statement conforms with the writer's experience. As Q increases, the cone of depression is deepened. Similarly, if the radius of the well is decreased and Q remains constant, the cone of depression is deepened. The effect of K on the cone of depression is well known. It is probably the most important factor in determining the well characteristics. An interesting feature of this parameter is that the shape of the cone of depression is a function of the reciprocal of the square of the radius of the well. This seems unusual because it is usually believed that the radius of the well has a relatively small effect on the well characteristics. However, the radius of the well does affect the shape of the cone of depression in the zone near the well because it is here that the slope of the free surface is greatest.

The author is to be commended for the contribution he has made to the knowledge of the flow of water into wells. The solution of many problems involving the flow of water through porous media requires the use of mathematical techniques which older engineers never had the opportunity to study. It is gratifying to note that the younger engineers are applying their knowledge to the solution of these problems which have defied rigorous analysis in the past.

DAVID K. TODD[15] AND LLOYD C. FOWLER,[16] JUNIOR MEMBERS, ASCE.—
The problem of finding an accurate and rational solution for gravity flow into a well has been of interest to engineers for many years. The model studies and analytic studies reported by Mr. Hansen are valuable contributions to this field. This discussion embodies a suggestion to extend the method of analysis developed by the author.

The interpretation of the dimensionless parameter, in terms of the ratio of the Froude number to the Reynolds number, indicates certain inconsistencies in the interpretation of the definitions involved in these dimensionless quantities. The Reynolds number, which is commonly used to define the influence of viscosity on the flow pattern, is normally expressed as

$$\mathbf{R} = \frac{\rho v l}{\mu} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (30)$$

in which ρ is the density, v the velocity, μ the viscosity of the fluid, and l is a characteristic linear dimension. For flow in pipes, the diameter of the pipe is used for l, and for flow in a porous medium, some measure of the pore openings or grain size is used.[17]

The author defines v' as $\left(\frac{Q}{C\,d^2}\right)$. If d represents a characteristic grain size or pore size of the porous medium, and if the dimensionless constant C combines the geometrical shape factor of the internal structure of the porous

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[17] "The Flow of Homogeneous Fluids Through Porous Media," by M. Muskat, McGraw-Hill Book Co., Inc., New York, N. Y., 1946, p. 56.

medium with the number of interstices through which flow is occurring, then v' is the mean microscopic velocity of flow through the aquifer.[18] The combination of this velocity with the well radius as the characteristic length in the Reynolds number, as done in Eq. 14*a*, has no relation to defining the flow conditions. To illustrate this, two horizontal pipes, filled with sand under identical conditions, are assumed to be connected, at the same elevation, to a constant-head tank. According to the definition of the Reynolds number, as given in Eq. 14*a*, for the first pipe,

$$\mathbf{R}_a = \frac{\rho_a v_a r_a}{\mu_a} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (31)$$

and for the second pipe,

$$\mathbf{R}_b = \frac{\rho_b v_b r_b}{\mu_b} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (32)$$

in which r_a and r_b are the radii of the first and second pipes, respectively. Under any given temperature conditions, $\rho_a = \rho_b$ and $\mu_a = \mu_b$, and, since both pipes are under the same pressure head, $v_a = v_b$ by the Darcy law. Dividing $\mathbf{R}_a$ by $\mathbf{R}_b$, and canceling terms, the ratio of the two Reynolds numbers is

$$\frac{\mathbf{R}_a}{\mathbf{R}_b} = \frac{r_a}{r_b} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (33)$$

If the first pipe has twice the radius of the second pipe, this does not mean that $\mathbf{R}_a = 2\,\mathbf{R}_b$, but it does signify that the flow conditions are identical in the two pipes, and, hence $\mathbf{R}_a$ must equal $\mathbf{R}_b$. This result can be achieved only by replacing the pipe radii with the characteristic pore or grain sizes of the sand. These sizes will necessarily be equal for the same sand and packing conditions.

The same reasoning can be applied to the Froude number to show that r_w is not the characteristic length used to determine v'. To obtain the author's ratio, it would be necessary to express $\frac{Q}{K r^2_w}$ as $\frac{Q}{K d^2}$. The variable d does not depend on the well characteristics, but is a property of the porous medium only. Thus, in Eq. 12, K is defined in terms of d.

The author's statement that the parameter $\frac{Q}{K r^2_w}$ is "an index of the shape of the cone of depression around the well" can be illustrated by reference to Eq. 6. Rewriting this equation as

$$Q = 2\pi r h K \frac{dh}{dr} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (34)$$

and substituting the parameter yields

$$\frac{Q}{K r^2_w} = 2\pi \left(\frac{h_s}{r_w}\right)\left(\frac{dh}{dr}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (35)$$

in which $r = r_w$ and $h = h_s$. This may be stated as follows:

$$\frac{Q}{K r^2_w} = C \left(\frac{h_s}{r_w}\right) \tan \alpha \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (36)$$

[18] "The Theory of Ground-Water Motion," by M. K. Hubbert, *Journal of Geology*, Vol. 48, 1940, p. 816.

in which C is a constant, and α is the angle between the horizontal and the free water surface at the well. The value $\frac{h_s}{r_w}$ may vary from zero to twenty-five, as shown in Fig. 13, whereas tan α may vary from zero to infinity. Thus, the slope governs the magnitude of the parameter, a large value indicating a deep cone and a small value indicating a shallow cone.

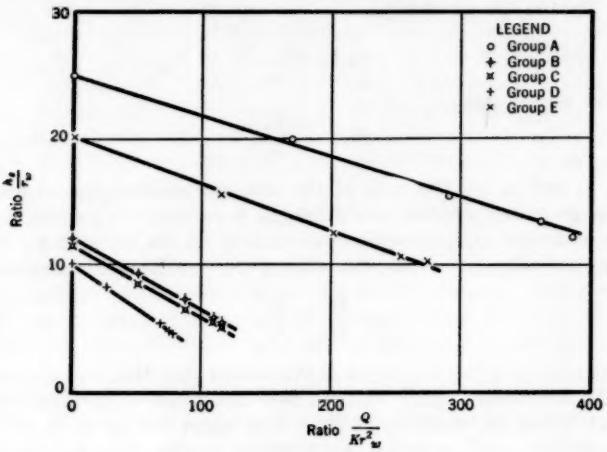

FIG. 13.—RELATION OF $\frac{h_s}{r_w}$ TO $\frac{Q}{K r^2_w}$

Because of a typographical error in the paper by Messrs. Babbitt and Caldwell,[5] Mr. Hansen's statement that their "* * * maximum external radius was only 25 well radii * * *" is in error. The actual maximum external radius can be found from the values, obtained by Messrs. Babbitt and Caldwell, for $\frac{h_e}{r_e}$ and $\frac{r_w}{h_e}$ as follows:

$$\frac{h_e}{r_e} \times \frac{r_w}{h_e} = \frac{r_w}{r_e} = 0.155 \times 0.040 = 0.0062 \ldots\ldots\ldots\ldots (37)$$

This relationship yields

$$r_e = 161.3\, r_w \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (38)$$

An examination of the author's analysis of gravity flow into a well and of data presented by the author and Messrs. Babbitt and Caldwell (tabulated in Table 1) indicates that it may be possible to extend the relationships involved by introducing two new symbols. The first symbol is the variable h_e, denoting the piezometric head at the well when there is no drawdown, and a β-factor, which is a measure of the resistance to flow at or near the well face.

TABLE 1.—SUMMARY OF DATA FROM MODEL TESTS

Group	Test No.	$\frac{h_s}{r_w}$	$\frac{h_s}{r_w}$	$\frac{h_w}{r_w}$	$\frac{Q}{K r^2_w}$	$\frac{h_s{}^{(a)}}{h_e}$	Factor[b]
A[c]	1	25.00	20.00	18.75	169		0.0317
	2	25.00	15.62	12.50	290		
	3	25.00	13.50	6.25	362		
	4	25.00	12.25	0.00	386	0.490	
B[c]	5	12.06	9.28	9.04	50.9		0.0566
	6	12.06	7.28	6.03	86.9		
	7	12.06	5.79	3.02	109		
	8	12.06	5.43	0.00	116	0.450	
C[c]	9	11.50	8.51	8.62	50.7		0.0565
	10	11.50	6.44	5.75	87.1		
	11	11.50	5.52	2.88	109		
	12	11.50	5.06	0.00	116	0.440	
D[d]	1	10.1	8.27	8.23	26.1		0.0706
	5	10.1	5.40	3.60	67.8		
	7	10.1	4.73	1.66	75.5		
	8	10.1	4.60	0.28	77.4	0.455[e]	
E[f]	1	20.00	15.6	15.2	115		0.0367
	2	20.00	12.6	10.2	202		
	3	20.00	10.7	5.2	254		
	4	20.00	10.3	0.0	273	0.515	

[a] When $h_w = 0$. [b] Average of the four tests in each group. [c] Babbitt and Caldwell, electrical model. [d] Babbitt and Caldwell, sand model. [e] Estimated value. [f] Hansen, sand model, from Fig. 10.

Messrs. Babbitt and Caldwell[5] found that Q was a linear function of h_s; therefore, in terms of dimensionless parameters, an equation of the form:

$$\frac{h_s}{r_w} = -\beta \frac{Q}{K r^2_w} + \frac{h_e}{r_w} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (39)$$

may be used to relate these parameters. The constant term $\frac{h_s}{r_w}$ appears on the right side of the equation so that when $Q = 0$, indicating no drawdown at the well, Eq. 39 becomes $h_s = h_e$. The negative sign is inserted in the equation because of the inverse relation of Q to h_s. To verify Eq. 39, values of $\frac{h_s}{r_w}$ and $\frac{Q}{K r^2_w}$ were plotted in Fig. 13. These points were fitted by straight lines having an intercept value of $\frac{h_e}{r_w}$, and it is seen that these lines fit closely the observed data, thus verifying the findings of Messrs. Babbitt and Caldwell. The slopes of these five lines, represented by the β-factor in Eq. 39, are tabulated in Table 1. If the β-factor were increased for any one of the sets of data, the slope of the line through the fixed intercept would be increased negatively; hence, for a given value of $\frac{Q}{K r^2_w}$, $\frac{h_s}{r_w}$ would be less. This means that a greater drawdown would be required to obtain the same discharge since K and r^2_w are constant. Thus, the β-factor is directly proportional to flow resistance. More extensive data may make it possible to relate the β-factor to types of well screens or perforation, to gravel packing conditions, or to the degree of clogging of the openings in the well face. The deterioration of a well with age might be expressed in terms of increasing values of the β-factor.

Completion of the analysis necessitates relating $\frac{h_s}{r_w}$ to $\frac{h_w}{r_w}$, as is done in Fig. 14, based on data from Table 1. The curves through these points extend from the line having the equation,

$$\frac{h_s}{r_w} = \frac{h_w}{r_w} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (40)$$

when $Q = 0$ to the line $h_w = 0$, when Q is a maximum. It can be seen that the curves are all of the same general form, indicating that a general mathematical

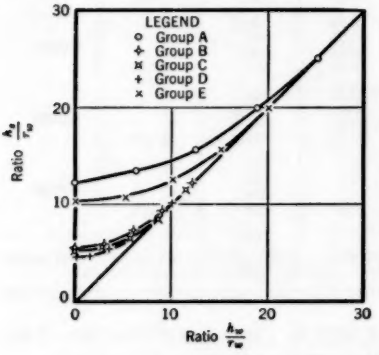

FIG. 14.—RELATION OF $\frac{h_s}{r_w}$ TO $\frac{h_w}{r_w}$

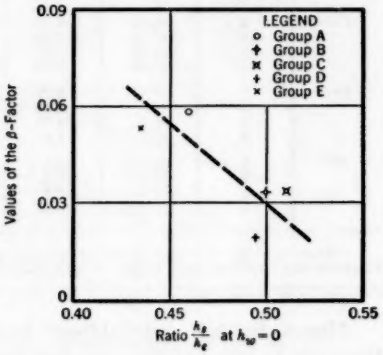

FIG. 15.—RELATION OF β-FACTOR TO $\frac{h_s}{h_e}$, WHEN $h_w = 0$

relationship can be developed for the family of curves if the end points are known. The end point on the line of Eq. 40 is readily obtainable because at this point

$$\frac{h_s}{r_w} = \frac{h_w}{r_w} = \frac{h_e}{r_w} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (41)$$

and $\frac{h_e}{r_w}$ is known. To find the value of $\frac{h_s}{r_w}$, when $\frac{h_w}{r_w} = 0$, requires another step. A solution can be obtained if the relative drawdown outside the well, when $h_w = 0$, is assumed to be some function of the resistance to flow in the vicinity of the well face. In Fig. 15, the β-factor, expressing this flow resistance, is plotted against the dimensionless parameter $\frac{h_s}{h_e}$ using the data recorded when $h_w = 0$. The curve through the plotted points indicates a monotonic relationship, so that as the β-factor increases, the ratio $\frac{h_s}{h_e}$ decreases. Therefore, a well with a great flow resistance will have a larger relative drawdown, and a smaller discharge, when $h_w = 0$, than will a well with a small resistance. Using the relationships of Figs. 14 and 15, it is thus possible to relate $\frac{h_s}{r_w}$ to $\frac{h_w}{r_w}$. No attempt has been made to derive a general equation for this rela-

tionship because of the limited amount of data available. Since the data in Table 1 are from laboratory experiments on electrical and sand models, additional field data are necessary to verify, refine, and extend the relationships presented.

It has been assumed throughout this discussion that the relationships apply only to steady-state flow conditions. Under field conditions, the time lag between the initiation of pumping and the development of steady-state conditions may amount to several days. L. K. Wenzel[19] has shown that the drawdown curve in a pumping test in the Platte River Valley, in Nebraska, was still falling after 48 hours of continuous pumping.

To apply the relationships presented, one of the three variables h_s, h_w, or Q is assumed. From this assumed value, the values of the other two variables can be found, thereby providing for the determination of all the well variables. Such solutions, however, are predicated upon knowing the constants K and h_e, and being able to determine, from the well's history, construction, and structure, the constants r_w and β. It is also necessary that a linear relationship exist between h_s and Q, and that the ratio $\frac{h_s}{h_e}$, when $h_w = 0$, be uniquely defined in terms of the β-factor.

If additional data substantiate this extension of the author's analysis, coaxial graphs or nomographs can be prepared for a rapid evaluation of well variables. This analysis also suggests the interesting possibility of determining the radius of influence in the Dupuit equation from variables at the well.

VAUGHN E. HANSEN,[20] J.M. ASCE.—The writer wishes to express his sincere appreciation to the discussers. They have strengthened many of the concepts presented in the paper by adding their experience and analysis. The science of the flow of water into unconfined wells is such that engineers must work together, each contributing a segment to the whole, in order to arrive at a valid solution.

The discussion of the limitation of the Darcy law, presented by Mr. Shukry, is not clear. It is general practice to consider the validity of the Darcy law in terms of the Reynolds number of the flow that encompasses implicitly the gradient and permeability referred to by Mr. Shukry. Mr. Muskat[3] defines this limit as follows:

> "* * * it will suffice to accept as a safe lower limit where deviations from Darcy's law will become appreciable as given by a Reynolds number of 1, with d chosen as any reasonable average diameter of the sand grains."

Mr. Shukry states that

> "Because the Dupuit value of the slope m as given by Eq. 23 is not constant, it follows that the logarithmic functions shown in Fig. 7 cannot be straight lines."

Such is not the case because the piezometric heads in Eqs. 3 and 8 are not the same. The same symbols are used to avoid a complex symbol notation, but

[19] "Specific Yield Determined from a Thiem's Pumping-Test," by L. K. Wenzel, *Transactions*, Am. Geophysical Union, Vol. 14, 1933, p. 475.

[20] Irrig. Engr., Irrig. Div., SCS; and Associate Prof. of Research, Utah State Agri. College, Logan, Utah.

the reader should note carefully that for the Dupuit equation (Eq. 3) to be valid, the measurements of piezometric head (h) should be made in zones of essentially horizontal flow, whereas Eq. 8 applies to measurements taken in the curvilinear zone at or near the free surface. Such limitations result in Eq.23 being an approximation that consequently affects the conclusions drawn therefrom.

Mr. Boulton's finding that "* * * the drawdown ($h_e - h_s$) at the well, is practically independent of the water level in the well for ranges of h_w/h_e between zero and 0.4" is mentioned by Mr. Shukry, who suggests that the writer check this conclusion. Fig. 10 shows that, even for a change in values of h_w/r_w from 0 to 6 corresponding to a change of h_w/h_e from 0 to 0.37 for a $\left(\frac{Q}{K\,r^2_w}\right)$-value of 250, the change in h_s/r_w is relatively small, being approximately 12%. However, the drawdown continued to increase until h_w was equal to 0. A further insight into the variation of h_s with a variation in h_w is given in Fig. 14 in which h_s/r_w and h_w/r_w are plotted. The significance of the change in h_s for a change of h_w/h_e from 0 to 0.4 depends on the application; a change of 12% in drawdown may be important in certain applications. Hence, the writer would prefer stating that the drawdown does not increase greatly when h_w/h_e is reduced to less than 0.4. However, the drawdown does continue to increase until $h_w = 0$.

Eq. 27, solved for the proper boundary conditions, should yield the desired solution to the shape of the free surface. However, the application of the proper boundary conditions is a complex problem. The simplified boundary conditions applied by Mr. Shukry (at $r = r_w$, $h = h_s$ and $dh/dr = \infty$) to obtain Eqs. 28 and 29 are boundary conditions at a point and not the boundary conditions that apply to Eq. 21. Since Eq. 21 involves the total flow, the applied boundary conditions must be those governing the total flow. The solution obtained, as represented by Eq. 29, is a solution for a well having Mr. Shukry's boundary conditions along the entire inner boundary. A detailed discussion of the boundary conditions at an unconfined well has been presented by Chong-Hung Zee[21] in a doctoral dissertation at the Utah State Agricultural College, in Logan.

Further proof that Eq. 29 is incorrect is obtained when both sides of the equation are divided by r_w:

$$\frac{h_s}{r_w} = \frac{1}{2\,\pi}\left(\frac{Q}{K\,r^2_w}\right) \qquad (42)$$

This equation is seen to lack the term h_w/r_w of Eq. 11. Reference to Fig. 10, in which the parameters of Eq. 11 are plotted, shows a family of curves designated as h_w/r_w. Hence, both Eq. 11 and Fig. 10 prove that h_s is not independent of h_w as claimed by Mr. Shukry.

Mr. Rohwer, in discussing the effect of the discharge number, $\frac{Q}{K\,r^2_w}$ (and particularly the radius of the well, r_w) on the cone of depression, comments that it is surprising

[21] "The Use of Combined Electrical and Membrane Analogies to Investigate Unconfined Flow into Wells," by Chong-Hung Zee, thesis presented to the Irrig. and Drainage Dept., Utah State Agri. College, in Logan, Utah, in 1952, in partial fulfilment of the requirements for the degree of Doctor of Philosophy.

"* * * that the shape of the cone of depression is a function of the reciprocal of the square of the radius of the well. This seems unusual because it is usually believed that the radius of the well has a relatively small effect on the well characteristics."

The discharge number is deceiving in that one may be tempted to state that the discharge will vary as the square of the radius. However, observing Eq. 11, it will be seen that the radius also appears on the right side of the functional relationship. To clarify the effect of a change of radius on the discharge of the well, Fig. 16 has been prepared from theoretical Eqs. 2and 3 for flow into both confined and unconfined wells. A family of curves exists since a change in well radius will usually produce a change in external boundary conditions.

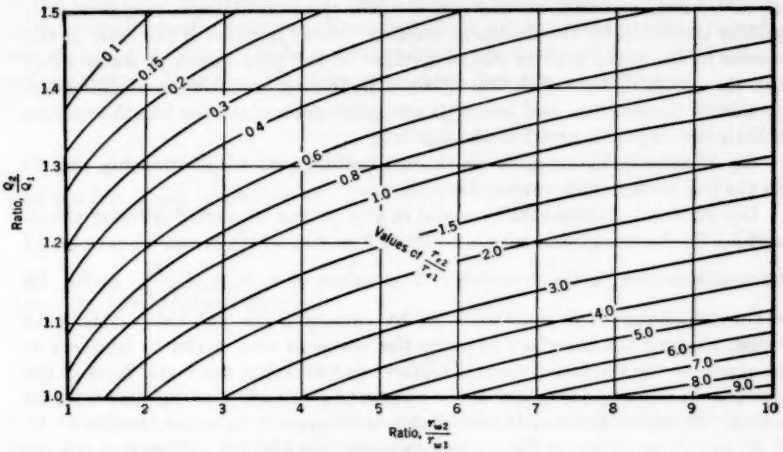

FIG. 16.—RELATIONSHIP BETWEEN DISCHARGE AND WELL RADIUS FOR EITHER CONFINED OR UNCONFINED FLOW.

However, if the external conditions remain the same, an increase of 100% in well radius only produces an increase of approximately 11% in the well discharge. Hence, as far as the general hydraulics of the flow are concerned, Mr. Rohwer is correct in feeling that the radius of the well has a relatively small effect on the well characteristics—particularly on the discharge. However, practical considerations not encompassed by the theoretical formula often play an important part in the relation between the discharge and the well radius. Since factors affecting loss of energy at and in a well are more important in a small well than in a larger one, losses at the well frequently reduce the discharge materially. Fig.16 summarizes the effect produced by a change of well radius on the discharge from an ideal well in which the losses at and in the well are negligible.

The writer disagrees with Messrs. Todd and Fowler in their discussion of definitions of the Froude and Reynolds numbers involved in the discharge number. Those discussers assert that combining the mean microscopic velocity of the flow "* * * with the well radius as the characteristic length in the

Reynolds number, as done in Eq. 14*a*, has no relation to defining the flow conditions." The mean grain diameter, or some other representative cross-sectional measure of the flow path, is generally used to evolve a Reynolds number that is characteristic of the flow through porous media. Flow to a well is certainly flow through porous media, but one subtle difference becomes apparent as one carefully analyzes the implication of the discharge number. This number is a characteristic of flow into a well, not flow through porous media; consequently, the characteristic length in the Reynolds number should be related to the geometry of the well—otherwise the derived Reynolds number will have no relation to the well. The writer's analogy, as involved in Eqs. 31, 32, and 33, is correct and sound for flow in porous media. Such a discussion has no bearing, however, on flow into a well if the cone of depression is quantitatively described by the discharge number (which in effect is the ratio of the viscous forces to the gravity forces involved in the phenomona of water entry into the well). The well is the internal boundary condition controlling the drawdown phenomona, and hence its geometry determines the length criterion in both the Reynolds and Froude numbers.

Eq. 38 extends the range to which the linear logarithmic relationship (Eq. 8) fits the free surface with reasonable accuracy.

Eq. 39 is not fundamentally sound in that h_e can be varied without affecting h_s. To illustrate, reference is made to Fig. 4 in which h_e can be considered the water surface in the reservoir. The values of h_s/r_w and $\frac{Q}{K r^2_w}$ in Eq. 39 would not change if h_e were reduced by removing the left half of the sand model, shaping the interface between the reservoir and model to conform to the shape of the original potential surface, and lowering the water level in the reservoir to the original height of the measured free surface at the new external radius. A correct functional relationship is obtained if h_w is combined with h_s, Q, K, and r_w as shown in Eq. 11 and as plotted in Fig. 10. Note that the coordinates of Fig. 10 are identical to those of Fig. 13.

It is questionable whether the β-factor suggested by Messrs. Todd and Fowler is a measure of the resistance to flow at or near the well face and hence indicative of the type of well screen, perforation, gravel pack, or degree of clogging, as they suggest. Analysis of the β-factor in Table 1 and in Fig. 15 indicates that it is a direct function of the applied boundary condition and consequently will not be a function of well resistance or well losses. It appears that the points are plotted incorrectly in Fig. 15. This error may be easily rectified by reference to Table 1 in which the correct values are listed.

The results of the North Platte River Valley (Nebraska) pumping tests[19] were pointed out by Messrs. Todd and Fowler to show that generally steady-state flow conditions are not reached for several days. Actually, steady-state conditions are never reached unless the water table is sloping, or unless vertical recharge is involved. Modifications of steady-state flow formulas to the cases of sloping water tables and vertical recharge are presented by Dean F. Peterson, Jr., Orson W. Israelsen, Members, ASCE, and the writer.[22] Steady-state flow formulas also can be applied to unsteady horizontal flow. The interested

[22] "Hydraulics of Wells," by Dean F. Peterson, Jr., Orson W. Israelsen, and Vaughn E. Hansen, *Technical Bulletin No. 351*, Agri. Experiment Station, Utah State Agri. College, Logan, Utah, March, 1952.

reader can use the Nebraska tests, referred to by Messrs. Todd and Fowler, to verify the contentions that follow.

The analysis of unsteady unconfined flow involves two types of problems. The first type of problem applies to measurements of piezometric head taken in the zone where the Dupuit equation applies ($h^2 \approx \log r$). In this case the square of the piezometric head (h^2) should be plotted against the logarithm of the radius of the well. The second type of problem applies to measurements taken in the zone of curvilinear flow near the free surface where the linear logarithmic relation applies ($h \approx \log r$). This second case also applies to the analysis of unsteady confined flow to wells. Making the appropriate semi-logarithmic graph will show the plotted data adjacent to the well lying in a straight line regardless of the time since pumping began. The extent of the straight line will increase as the pumping test continues. The slope of the resulting straight line, regardless of its radial extent, will be a valid index to the discharge and coefficient of permeability. Note that Eq. 3 for unconfined flow can be written as

$$\frac{h^2 - h_1^2}{\log_e \frac{r}{r_1}} = \frac{Q}{\pi K} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (43)$$

whereas Eq. 8 can be written as

$$\frac{h - h_1}{\log_e \frac{r}{r_1}} = m \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (44)$$

Also, Eq. 2 for confined flow is

$$\frac{h - h_1}{\log_e \frac{r}{r_1}} = \frac{Q}{2 \pi K t} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (45)$$

The left sides of Eqs. 43, 44, and 45 are the slopes of the straight lines resulting from the appropriate semi-logarithmic plotting.

A further extension of the foregoing analysis will provide a sound quantitative evaluation of the radius of influence. It is suggested that the radius of influence be considered as the radial distance to the intersection between the straight line as described and the original water table, or piezometric surface, before pumping was begun.

Additional experimental data concerning flow to unconfined wells can be found in Mr. Zee's dissertation which analyzes the free surface in terms of the dimensionless parameters presented herein by combining his own data with those of Messrs. Babbitt and Caldwell,[5] Yang,[11] Zee[21] and that of the writer.[23] Mr. Zee obtained his results by combining the electrical analogy with the membrane analogy suggested by the witer.[24] Using the data of these same men, a further extension of well hydraulics has been made by Messrs. Peterson, Israelesen, and the writer.[22]

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[23] "Evaluation of Unconfined Flow to Multiple Wells by the Membrane Analogy," by Vaughn E. Hansen, thesis presented to the State University of Iowa, at Iowa City, Iowa, in 1949, in partial fulfilment of the requirements for the degree of Doctor of Philosophy.

[24] "Complicated Well Problems Solved by the Membrane Analogy," by Vaughn E. Hansen, *Transactions*, Am. Geophysical Union, Vol. 33, No. 6, December, 1952, p. 912.

LLOYD E. MYERS, JR.,[3] J. M. ASCE.—The large quantities of water required for rice irrigation are a cause for great concern in any water-deficient area in which rice irrigation is practiced. The quantity of water used for rice production in any given field is governed by a number of factors, including the cultural practices utilized by the rice grower concerned. Two factors mentioned by Mr. Shutts are land leveling and spacing of contour levees. A third factor worthy of mention is the practice of draining certain rice fields, at some time subsequent to the initial flooding, and reflooding those fields after accomplishing the purpose for which drainage was done. Although this practice is not followed by every rice grower, and in certain areas, may not be used at all, it is commonly utilized in enough areas within the United States to make it of general interest and concern. Such flood-drain-reflood cycles not only result in an increased use of irrigation water, but also have a detrimental effect on human health, comfort, and economy by increasing the production of pest mosquitoes. The importance of water conservation is common knowledge. The importance of rice-field mosquitoes may not be so well-known.

Rice fields, because of their ordinarily intense production of mosquitoes, are of considerable concern to the public health. Malaria and rice growing have been synonymous throughout history. The near-elimination of malaria in the United States has reduced the danger of anopheline malaria vector mosquitoes; but (1953), the problem of encephalitis (sleeping sickness) and the mosquito *Culex tarsalis*, which is the principal vector of this disease in the United States, has not yet been conquered. Also of concern are the pestiferous *Aedes* and *Psorophora* mosquitoes, which fly long distances and bite viciously, and thus seriously interfere with the comfort and mental health of both workers and residents in the rice growing areas. Livestock and poultry production are often reduced by these pest mosquitoes. There are records of cattle deaths which resulted from attacks by swarms of *Psorophora* mosquitoes.

Because *Anopheles* and *Culex* mosquitoes lay their eggs directly on the water, production of these mosquitoes may be slightly reduced by the flood-drain-reflood cycles. *Aedes* and *Psorophora* mosquitoes, however, lay their eggs on moist soil. The eggs usually are laid on recently drained soil and then hatch when the soil is reflooded. Each cycle of draining and flooding ordinarily produces a huge brood of these mosquitoes. The mosquitoes whose numbers may possibly be slightly reduced by the flood-drain-reflood cycles fly relatively short distances, usually less than one mile, whereas the mosquitoes whose numbers are greatly increased by the cycles fly relatively long distances. The net result is an increase in the number of mosquitoes produced in the rice fields involved, and an increase in the distance from those fields that man, livestock, and poultry will be subject to attack by mosquitoes. This result is highly undesirable.

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[3] Asst. San. Engr. (R), Communicable Disease Center, Public Health Service, U. S. Dept. of Health, Education, and Welfare, Atlanta, Ga.

There are a number of reasons for the use of the flood-drain-reflood cycle, all of them based on benefits to rice production. The principal reasons are as follows:

1. Certain rice-damaging aquatic insects are not easily controlled by chemical means. When a rice field becomes infested with these insects, control is often accomplished by draining the field. The field is reflooded after the ground surface has dried enough to eliminate the insects.
2. Some rice growers believe that rice seedlings develop best in a moist, nonflooded soil, and use the flood-drain-reflood cycle for planting. The soil is thoroughly moistened by flooding the field before planting, the seed is sown by airplane in the flooded field, and the field is drained shortly after planting to permit optimum development of the rice seedlings. The field is reflooded after the seedlings are well established.
3. Rice fields are sometimes drained to check the excessive growth of rice plants which occurs under certain conditions. Such growth may result in a late crop which will not fill well, or mature. Excessive growth can be checked by draining the affected field, and drying it until the rice plants actually suffer from a lack of moisture. The field is reflooded after the growth has been checked.

Regardless of any undesirable results of the flood-drain-reflood cycle, from the standpoints of water conservation and public health, the practice is justified by the benefits to rice production. Reduction in the use of the cycle can be brought about only by the development of alternate means of accomplishing the purposes for which the cycle is used.

The flood-drain-reflood cycle is but one example of a situation associated with rice irrigation which creates problems in water conservation and public health; there are many others. Unfortunately, answers have not yet been found for most of these problems and much work remains to be done. It is hoped that investigations will develop practical answers. The relationships between water conservation, public health, and rice-cultural practices create a definite need for cooperative action on the part of the various groups concerned. Each group should become familiar with the interests and activities of the other groups. Joint interest and efforts will result in more rapid and effective solution of mutual problems than will uncoordinated and independent efforts.

E. E. Shutts,[4] M. ASCE.—Sixty-six years of rice-irrigation experience in southwest Louisiana leads to the observation that mosquitoes normally do not breed or concentrate in large quantities in irrigated rice fields. This is caused by the fact that the water in the rice fields is not stagnant, but constantly replenished and circulated. The water containing fish life is pumped from surface streams through the canal system and into the fields themselves. The fish destroy the mosquito eggs and prevent their breeding.

Sea Marshes.—What has been previously stated concerning mosquito breeding in rice fields also applies to the band of deep sea marsh extending along the entire coast of southwest Louisiana. A heavy concentration of mos-

[4] Cons. Civ. Engr., F. Shutts' Sons, Lake Charles, La.

quitoes is always found on the ridges along the marsh and in the short grass marshes that are alternately wet and dried. In the middle of the deep sea marsh, small fish life prevents the breeding or concentration of mosquitoes. During the years when the mosquito plague is greatest, mosquitoes are found in the high places in the marshes and in the prairie grass where rice is not cultivated.

Rice is not grown in water-short areas; it is only cultivated where water is abundant. However, water is never wasted as the pumping and the handling of it is too costly. Irrigation water is taken from streams or is a mixture of well and stream water. Rice plants consume water constantly until ready to head, some water evaporates, and a very minute quantity of water seeps through the ground despite the shallow hardpan. Rice irrigation aids rivers and bayous in draining land, and drainage is necessary to remove stagnant water in order to overcome mosquito plagues.

From 1913 to 1953, either as a result of natural causes or from the benefits of public health services, rice areas in southwest Louisiana and Arkansas were free of malaria and sleeping sickness. There were only a few scattered cases of malaria in southwest Louisiana during 1953, whereas fifty years previously, before the land was extensively cultivated for rice, malaria was very common. In these early days of rice culture in this area, when rice was irrigated by the Providence system (using stagnant water), rice culture may have contributed to the breeding of mosquitoes. At the present time (1953) there are no epidemics of malaria and there is a very high standard of public health in southwest Louisiana. A publication concerned with the life history of mosquitoes states that:[5]

> "The necessity of a low oxygen concentration for the hatching of 'Aedes' mosquito eggs was reported by C. M. Gjullin, C. P. Hegarty and W. M. Bollen in 1941. They found that any method, chemical or physical, caused hatching when the oxygen content of the water was reduced. They concluded that bacteria or other organisms stimulated 'Aedes' eggs to hatch by reducing the oxygen content of the water flooding 'Aedes' eggs, which are normally layed on 'dry' ground. This contradicts the early belief that heat, cold, or drying, by causing a so-called conditioning or incubation period, was a necessary prelude to hatching."

The method of irrigation used in southwest Louisiana actually increases the oxygen content in the water. If this were not so, fish life would not abound in the irrigation canals and rice fields, and this accounts for finding mosquitoes concentrated largely on marsh ridges and in prairie grass rather than in the irrigated rice fields.

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[5] "Insects," *The Year Book of Agriculture*, U. S. Dept. of Agri., Washington, D. C., 1952, pp. 476–486.

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